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ENGINEERING PROPERTIES OF A PELAGIC CLAY

By

H. J. Lee

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NAVAL CIVIL ENGINEERING LABORATORY
Port Hueneme, California 93043

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ABSTRACT

Pelagic clay is a common ocean soil type covering over 30 percent of the seafloor. Two high quality box core samples of pelagic clay were obtained and subjected to index and engineering property testing. The results of tests on both cores were similar indicating little areal variability. The shear strength near the soil-water interface was about 1 psi, a relatively high value, and the soil was found to be virtually incompressible up to a compressive stress of about 4 psi. When remolded, the strength of the soil was significantly decreased, by as much as a factor of 6. The soil was also found to be very compressible beyond a stress of 4 psi. A technique was developed for using triaxial test results to derive shear strengths below the level of sampling. The resulting strength profile indicates a very gradual increase of strength with depth. Implications regarding the use of direct embedment anchors are discussed.

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INTRODUCTION

This report is the second of a series dealing with engineering properties of seafloor soils on a worldwide basis, the first report having dealt with Bering Sea and North Pacific Ocean soils (Lee, 1973a). In this series the major deep ocean soil types (calcareous ooze, diatomaceous ooze, radiolarian ooze, and pelagic clay) will be sampled and estimates of in-situ engineering behavior will be made, primarily on the basis of laboratory test results. Reports will be prepared periodically as sets of tests on particular soil types are completed. Ultimately a final report summarizing these reports and directed toward practical engineering problems will be prepared. It will include general correlations of engineering properties with either soil type or soil index properties for the various materials found in the world's oceans.

This report includes the results of tests which were performed on two high quality box core samples obtained from the Pacific Ocean between Hawaii and California. The locations of the cores are proprietary; however, the water depth in this general region averages about 15,000 feet. The sampling device was a spade-type box corer developed at the Scripps Institution of Oceanography with the support of the Navy Electronics Laboratory (Rosfelder and Marshall, 1967). The cores measured 8 inches by 12 inches in cross section and were about 20 inches long. The length to width ratio of 2.5 is excellent in terms of reducing disturbance (Hvorslev, 1948). The cores are identified as Box A and B.

The two cores contained a typical deep ocean soil, pelagic clay (often referred to as "red clay" in the literature). This material is known to cover about 40 million square miles (or about 33 percent) of the seafloor. It is usually brown in color and contains very fine grained material, much of which is thought to be wind-borne dust and volcanic ash. Organic material with the exception of such items as shark's teeth is missing. The clay accumulated very slowly (about 1 to 2 mm per 1,000 years) and often contains manganese nodules. A map illustrating locations where pelagic clay is found is given in Figure 1.

The samples were subjected to vane shear and water content testing upon their arrival at NCEL (less than one week after being taken from the seafloor). About ten 2.5-in. diameter cores were cut from the box cores and sealed for later triaxial and consolidation testing.

PROCEDURES

Accepted procedures (Lambe, 1951) were followed in the performance of index property testing. Consolidation tests were conducted in either an Anteus back-pressure consolidometer or the NCEL Specially-Developed Low Pressure (SDLP) consolidometer (Herrmann and others, 1972). Triaxial testing was conducted using a system assembled at NCEL for testing seafloor soils. Samples were back-pressured with seawater using a standard mercury pot pressure generation system (Bishop and Henkel, 1962). Triaxial cells were fabricated using the design developed at the University of California at Berkeley (Seed and others, 1964). Fresh water was used as a cell fluid. The samples were strong enough to be trimmed with a standard wire trimmer without introducing any apparent disturbance. Samples were encased in two rubber membranes separated by silicone grease to prevent osmosis between cell and sample water. Pore and cell pressures were measured with the same conventional strain gage transducer. Volume changes during consolidation were measured with an inverted, calibrated U-tube containing a volume of colored silicone oil. One of the triaxial test specimens was consolidated under K_o (no lateral strain) conditions. In this test, the axial load was increased and the confining pressure adjusted in such a way that the volume of fluid flowing out of the sample divided by the change in sample height equaled the initial sample cross sectional area.

RESULTS

Index Properties

Index properties measured included water content, grain density, grain size distribution, and Atterberg limits. The grain densities, sand-silt-clay percentages, and Atterberg limits, are given in Table I. Plots of the grain size distributions are given in Figure 2. Water contents were obtained within one week following the coring and again about one year later from one of the small cores which was taken from a box core and sealed. Since engineering property testing was conducted during the intervening period, the water contents measured before and after this testing serve as a check on the quality of sample sealing. The measured water contents are shown as a function of depth within each core in Figure 3. As may be seen there was little change in water content during the storage period. There is also little difference in water content between the two cores and little variation with depth in each core. This consistency is reflected in the other index properties as well.

Consolidation Characteristics

Consolidation testing is conducted to obtain two types of information: (1) parameters which can be used in calculating the settlements of structures and (2) the densities and vertical stresses of the soil below the core. The first type of information, if corrected for sample disturbance, is probably valid for engineering analysis since the time frame of laboratory testing is comparable to that of field loading. The second type of information is more doubtful because obtaining density and stress profiles from a relatively short-term test requires an assumption that sediments consolidate under the very long term conditions of sedimentation in the same way as they do in the

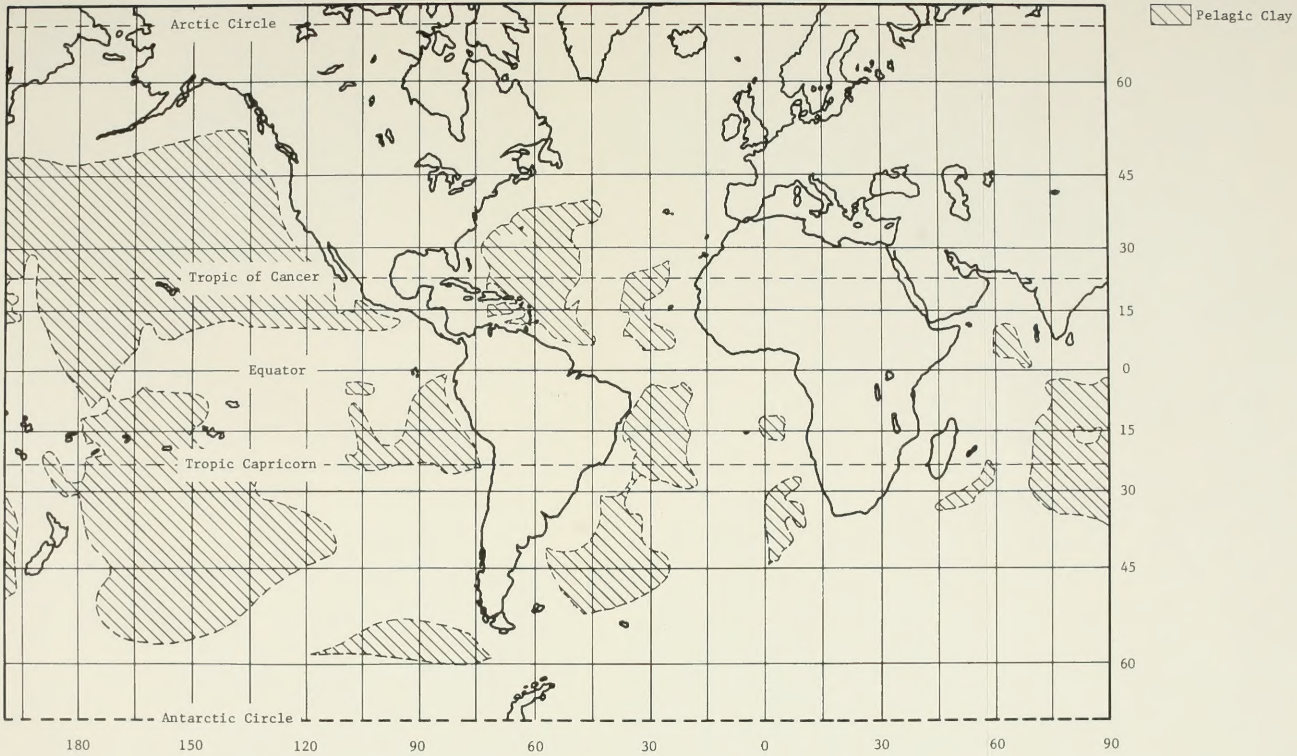

Figure 1. Distribution of pelagic clay in the oceans.
(after Demars and Anderson, 1971)

Table I. Index Properties

Core	Location in Core	Liquid Limit (Percent)	Plastic Limit (Percent)	Plasticity Index	Grain Density (g/cc)	Percent Sand*	Percent Silt*	Percent Clay*
A	Top	115	56	59	2.72	8	40	52
A	Bottom	124	55	69	2.72	6	38	56
B	Top	105	56	49	2.73	5	38	57
B	Bottom	109	61	48	2.69	4	39	57

* Sand>.05mm
.05mm>Silt>.002mm
.002mm>Clay

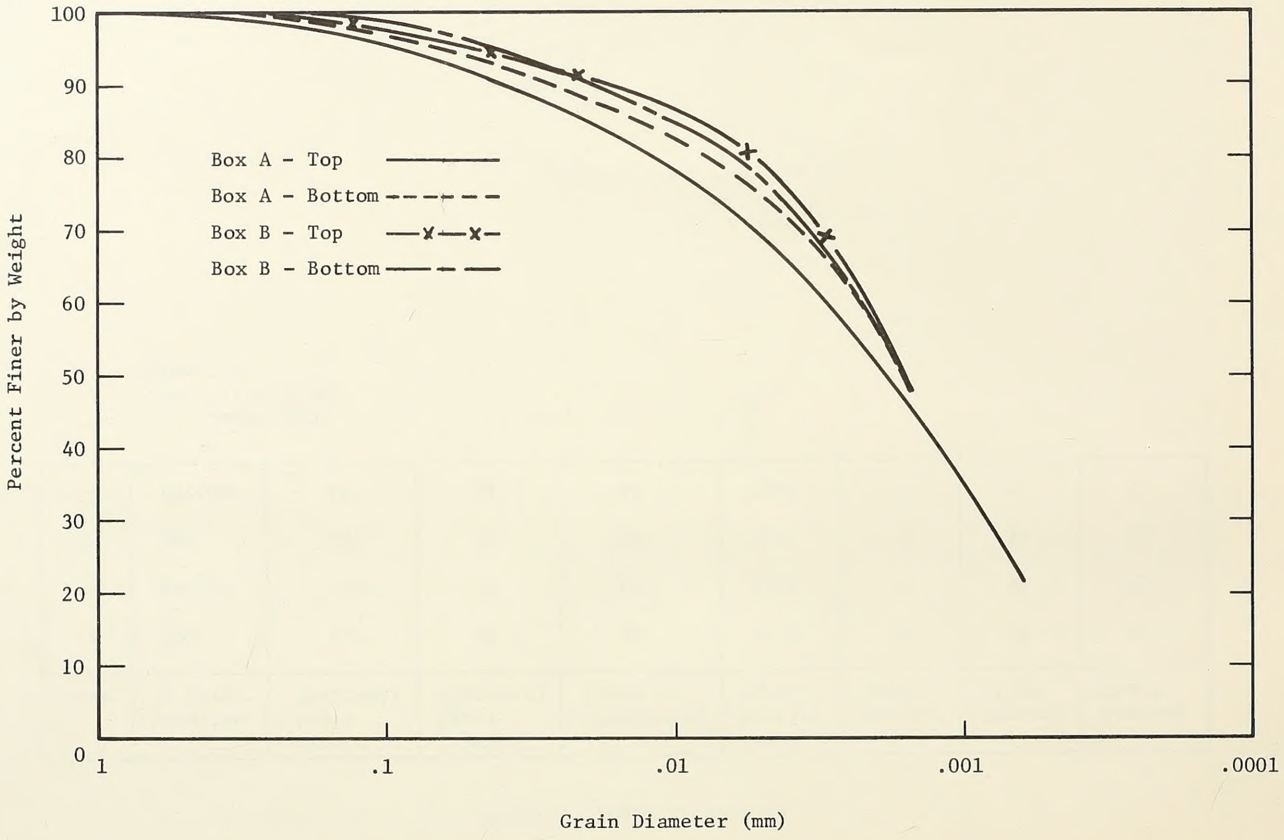

Figure 2. Distribution of grain sizes for both cores - hydrometer method.

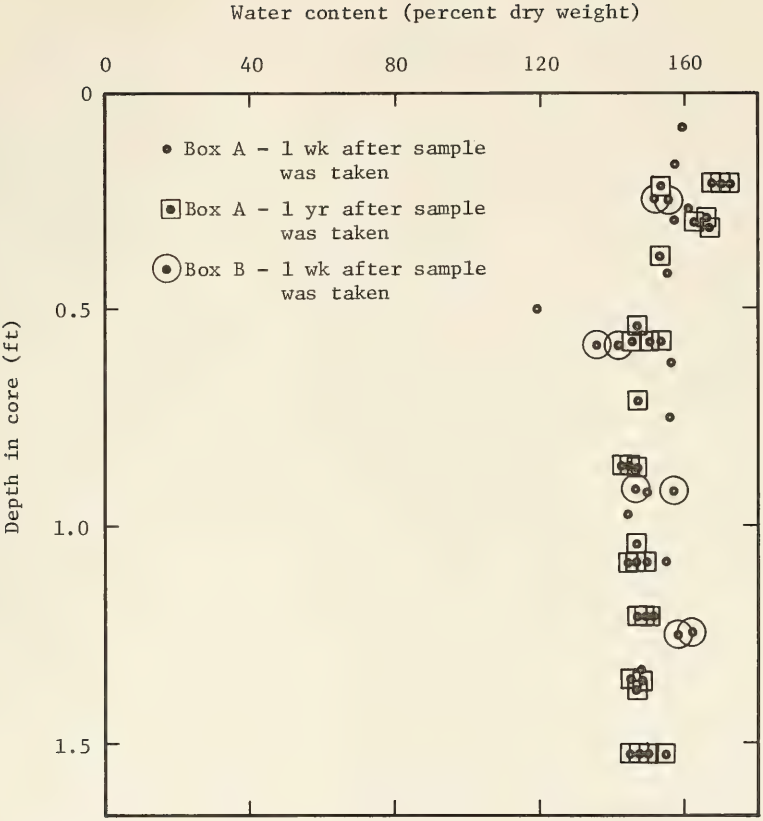

Figure 3. Water content distribution in both cores.

laboratory. This topic is currently being debated, and data are available to support almost any point of view. In this report it will be assumed that laboratory consolidation does approximately represent field sedimentation. Additional information on this subject will be obtained during a later phase of this project, and modifications to the conclusions of this report will be presented in subsequent reports, if necessary.

Four one-dimensional consolidation tests were conducted on specimens taken from the two box cores. Two tests were performed using a back-pressuring system (Anteus consolidometer) to increase saturation, and two were performed without back-pressure. No significant difference between the results of the two types of tests could be detected, thereby indicating that the samples were well saturated before testing.

The results of the tests are shown in Figure 4 in the form of the usual plot of void ratio versus the applied vertical effective stress. Three of the tests were performed on specimens from Box A and yielded almost identical results. Therefore, one average curve is presented for this box. Also shown on this figure are estimated "maximum past pressures" and estimated undisturbed consolidation curves.

The maximum past pressure, determined using the Casagrande (1936) technique, usually represents the maximum overburden pressure to which a soil has been subjected at some time in the past. For these pelagic clays the maximum past pressure is high (3.5 - 4.5 psi) relative to the current in-situ overburden stresses (0 - .3 psi). This could indicate a high degree of overconsolidation, produced perhaps by a scouring away of 20 to 30 feet of material. However, such a situation could also result from the development of relatively strong interparticle bonds near the sediment surface as a result of electrostatic or long-term factors. With no past embedment in excess of the current embedment, such a situation would be referred to as pseudo-overconsolidation. Whatever the cause of the behavior, the engineering applications of the results are approximately the same. The important aspect is that the material appears "overconsolidated" and as a result is much less compressible and, as will be seen, stronger than would be expected for a "normally consolidated" soil near the seafloor.

Estimated, undisturbed consolidation curves were constructed using the Schmertmann (1955) technique which was also used in the first report of this series (Lee, 1973a). The relatively close correlation between the undisturbed and laboratory consolidation curves is an indication of the high quality of the box core samples.

The usual static parameters obtained from these consolidation tests are given in Table II. The two compression indices (change in void ratio per one log cycle change in pressure) relate to whether the applied pressure is less than or greater than the "maximum past pressure." If it is less ("recompression"), the soil is almost incompressible. If it is greater (virgin compression), the soil becomes highly compressible. Obviously, engineering structures should be designed to apply pressures below the "maximum past pressure," if possible.

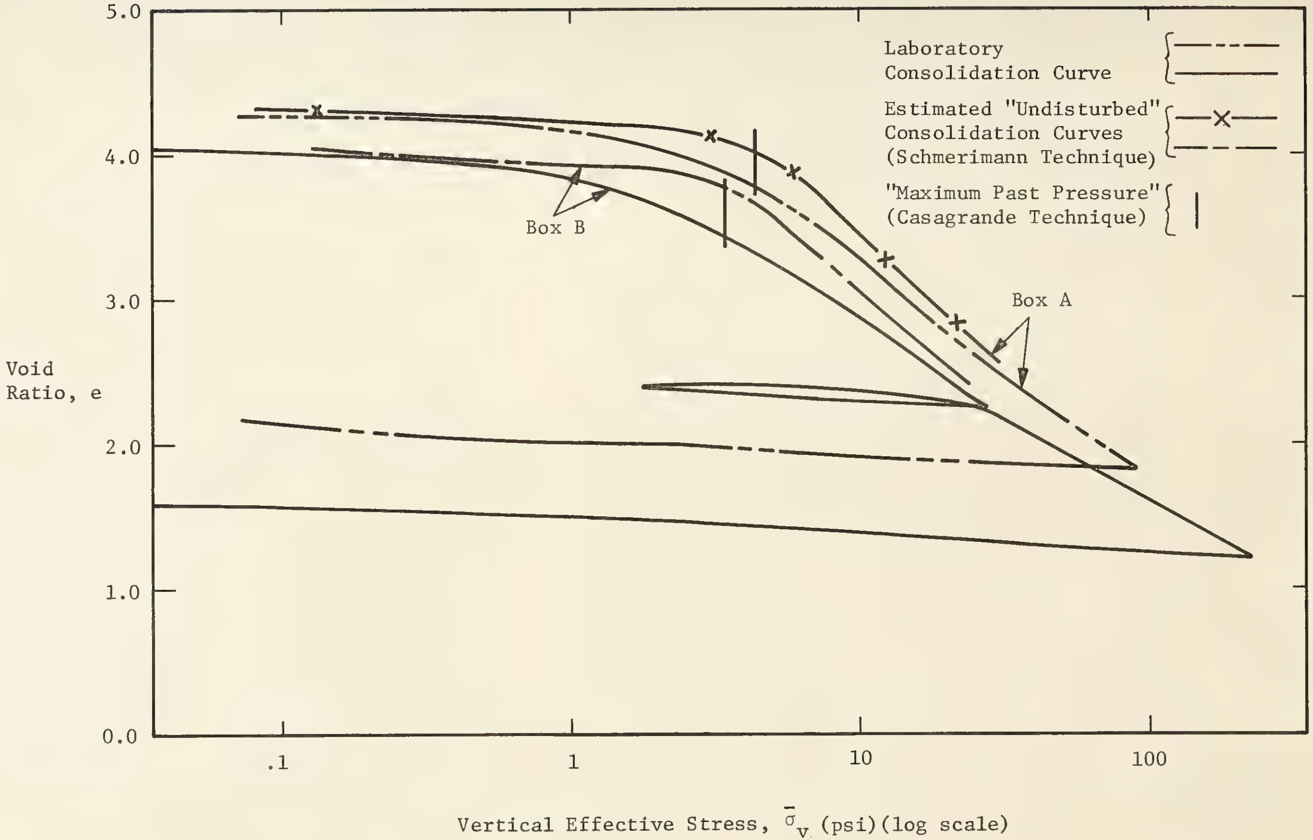

Figure 4. Stress-void ratio data from one-dimensional consolidation testing.

Table II. Static Consolidation Parameters

Core	In-Situ Overburden Pressure (psi)	"Maximum Past Pressure" (psi)	Recompression Index,* C_r	Virgin Compression Index,* C_c
Box A	0-.28	4.3	.12	1.82
Box B	0-.28	3.4	.15	1.70

* Compression Indices corrected for disturbance using Schmertmann (1955) technique.

Other data obtained from consolidation testing relates to time rate of consolidation. The two important parameters are the coefficient of consolidation, c_v, and the coefficient of secondary compression, C_α. Techniques for using both of these in settlement calculations are given in Navy Design Manual DM-7 (U. S. Navy, 1971). The coefficient of secondary compression, C_α, was obtained directly from plots of displacement versus the log of time. It is equal to the sample axial strain per log cycle of time for times clearly beyond the termination of primary consolidation (Lambe and Whitman, 1969, p. 421). The coefficient of consolidation, c_v, was calculated using the log time fitting method (Lambe and Whitman, 1969, p. 412). A plot of the calculated values of these parameters as a function of applied vertical effective stress is given in Figure 5. The values of c_v given are relatively low and the values of C_α are relatively high as compared with cohesive soils in general. In practical terms this means that these pelagic clays respond slowly to applied stresses and that significant movement may continue for a great deal of time into the future.

The distribution of vertical effective stresses and densities with depth below the seafloor may be estimated from consolidation data by assuming that laboratory compression behavior reproduces sedimentation behavior. Stresses are applied which simulate vertical overburden pressures at some depth. The soil density which the sample acquires is taken as the density at that assumed depth. The stress distribution may be obtained by converting the void ratios to densities and then integrating these densities incrementally with respect to sub-bottom depth. A plot of estimated vertical effective stress was obtained for Box A and is given in Figure 6. This information will be used in the next section to determine how the soil shearing strength increases with sub-bottom depth.

Shear Strength

Vane Shear. As soon as the box cores were received at NCEL, the box sides were removed and a number of vane shear tests were performed on the intact material. The tests were conducted with the vane axis perpendicular to the core axis. Small cylindrical cores were taken from the box samples and sealed for longer-term storage. Triaxial and consolidation tests were performed on these, and as the last phase of the test program, a series of vane shear and water content tests was performed on one of the small cores. A plot of all of the vane shear strength data as a function of depth in the core is given in Figure 7. There is considerable scatter to the data; however, the following conclusions appear justified.

1. There is little difference between the two cores.
2. The one-year storage had little influence on the strength.
3. There is no consistent trend of vertical variability, although there is some random scatter.
4. The mean vane shear strength for both cores is approximately 1 psi.

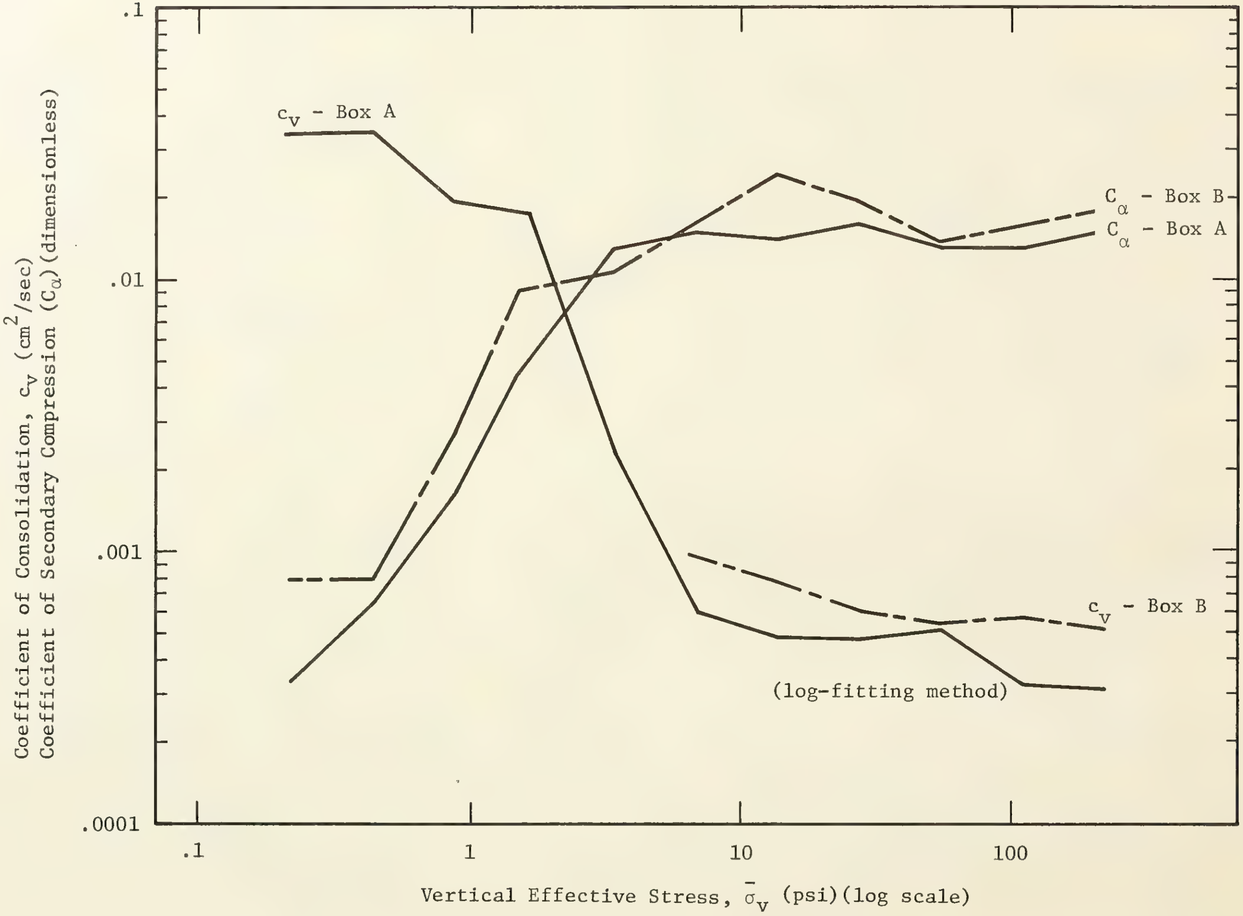

Figure 5. Coefficients of consolidation and secondary compression as a function of applied stress.

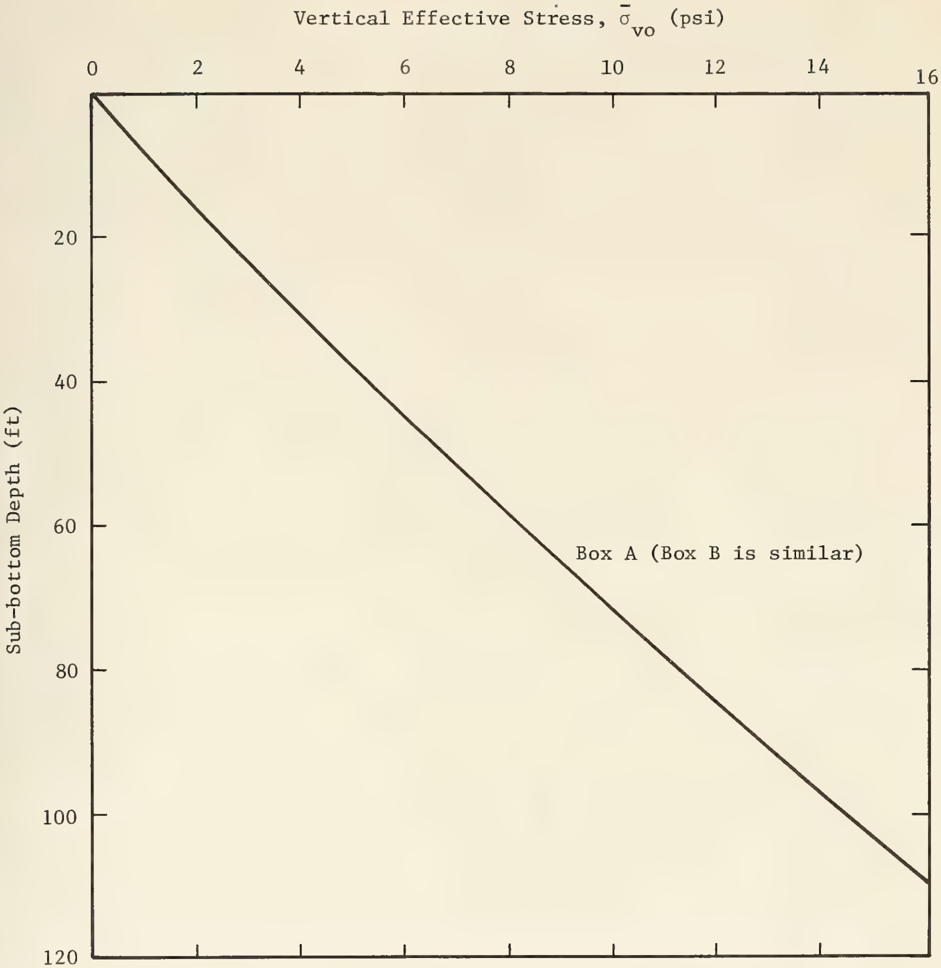

Figure 6. Estimated overburden stress, $\bar{\sigma}_{vo}$, as a function of sub-bottom depth.

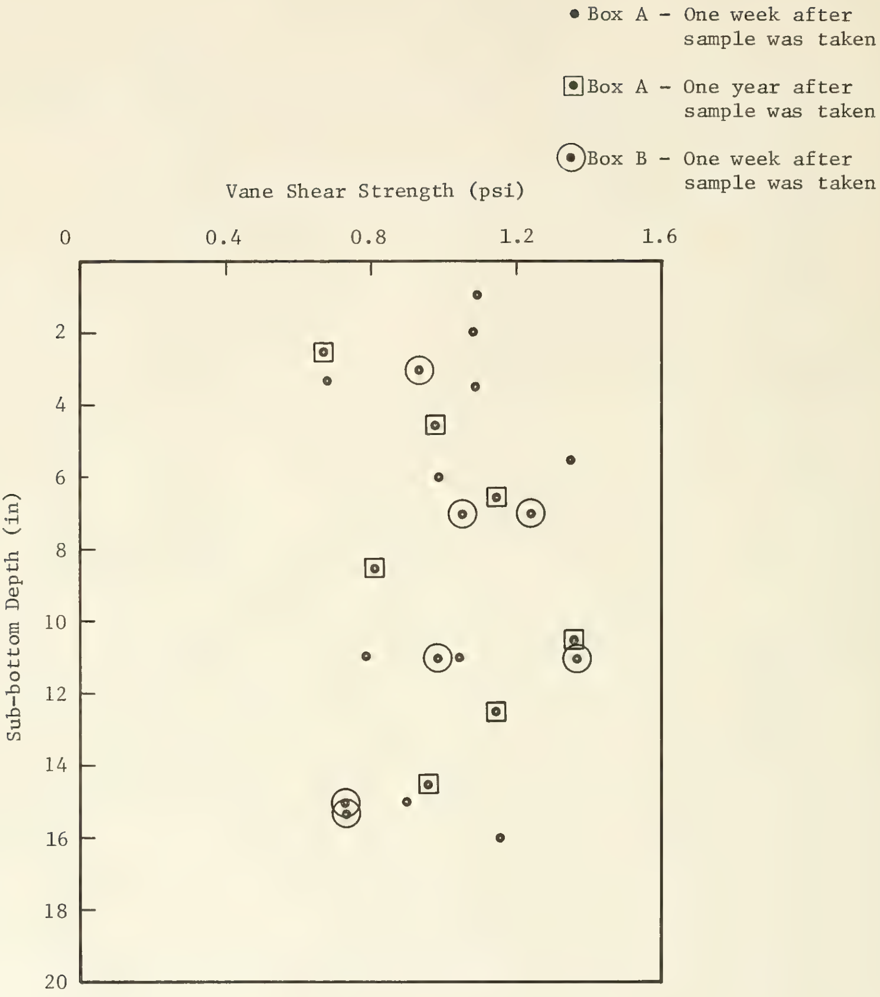

Figure 7. Vane shear strength distribution in both cores.

The first three conclusions are similar to those developed from the water content measurements. The last conclusion is significant. Strength values for surficial pelagic clays have been given in the literature (Keller, 1967) as being less than 0.5 psi. These earlier data were obtained from tests on small diameter gravity or piston cores. It appears likely that these corers disturb the sediment significantly, and that samples taken with them are much weaker than the material would be in-situ. The earlier results, therefore, appear to be incorrect. The in-situ strength of the box core material may be slightly higher than the laboratory strength, and it would be desirable to estimate the disturbance introduced during box coring using techniques involving residual pore pressure measurements. (Lee, 1973b). This is difficult, however, because certain disturbance parameters cannot be estimated on the basis of two-foot cores. When longer piston cores and in-situ data are available, an estimate of disturbance will be made. For future reference, the residual pore pressure retained by the box cores was measured and found to be equal to about -0.3 psi.

An estimate of soil sensitivity was made by rotating the vane rapidly through one revolution following a determination of the original strength. A second "remolded" strength is measured and a sensitivity is calculated as the ratio of original to remolded strength. The measured sensitivities varied considerably from a low of 3.1 to a high of 11.5. The mean sensitivity for both cores was about 6; this would classify the soil as a sensitive clay, requiring special attention during bottom operations.

Triaxial Shear. Five consolidated-undrained triaxial shear tests with pore pressure measurements were performed on carefully trimmed specimens obtained from Box A. Since the water content and vane shear tests indicated little difference between the two cores, it appears likely that the test results obtained are applicable to both cores and possibly to pelagic clays in general.

Triaxial testing has certain advantages over such simple tests as vane shear and unconfined compression. One advantage is that during the consolidation phase of the triaxial test it is possible to apply stresses which simulate conditions well below the level of sampling. It is, therefore, possible to estimate the variation of strength with depth. A second advantage of triaxial testing with pore pressure measurement is that it allows the determination of soil response in terms of effective stresses as defined by

$$\bar{\sigma} = \sigma - u \qquad (1)$$

where $\bar{\sigma}$ = effective normal stress
σ = total normal stress
u = pore water pressure

The vast majority of soil mechanics research indicates that it is the effective stresses which determine soil engineering behavior rather than the total stresses. Also effective stresses serve as a bridge between short-term (undrained) and long-term (drained) behavior as will be discussed later.

A triaxial test includes two distinct phases. During the first, the effective stresses of a certain location in the ground are applied, and the sample is allowed to consolidate (drain) until an equilibrium is reached. A soil specimen that would be taken from that location in the ground is approximately simulated in this manner. A problem again arises concerning the influence of time: the soil in-situ has taken millions of years to consolidate under the overburden while in the laboratory consolidation is accomplished in a few days. As discussed in the section on one-dimensional consolidation, the problem of time effects will not be approached in this report. It will be assumed that laboratory consolidation simulated field consolidation. Conclusions derived from this assumption will be modified if future research shows this assumption to be incorrect.

The second phase of a triaxial test involves closing the drainage lines to the sample and then gradually increasing the axial force until a certain strain (usually 20 percent) is reached. The test results are formulated in terms of effective stresses, a failure criterion is adopted, and the shear strength is estimated.

A concise way of expressing much of the information obtained from a triaxial test is through a stress path (Lambe and Whitman, 1969, p. 112). A stress path is a plot of one effective stress parameter versus another for the entire undrained shear portion of the triaxial test. The parameter $(\bar{\sigma}_1-\bar{\sigma}_3)/2$ is plotted versus $(\bar{\sigma}_1+\bar{\sigma}_3)/2$, where $\bar{\sigma}_1$ and $\bar{\sigma}_3$ are the major and minor principal effective stresses, respectively. In the NCEL tests the major principal stress was always equal to the vertical stress, and the minor principal stress, to the horizontal stress. The stress paths are defined in such a way that each point on the path represents the top point of a Mohr circle. A stress path is thereby a way of representing the infinite number of Mohr circles which correspond to the changing stresses occurring during a shear test.

The stress paths for the five pelagic clay triaxial tests are shown in Figure 8. Four of those were isotropically consolidated prior to shear, that is, the horizontal and vertical stresses were equal during consolidation. One of the tests (Number 5) involved anisotropic (K_o) consolidation prior to shear. The stresses were gradually increased in such a way that the sample average cross sectional area remained constant (no lateral displacement). Anisotropic (K_o) consolidation corresponds to the way soils compress in the field during sedimentation. It represents a state of shear, and the ratio of horizontal to vertical stress is identified as K_o, the coefficient of lateral earth pressure at rest.

The stress paths of Figure 8 appear to define relatively well a failure envelope. That is, the stress paths approach and then follow a particular line. Since the paths do not cross the line, it may be taken as a limiting stress condition or failure. The failure envelope is

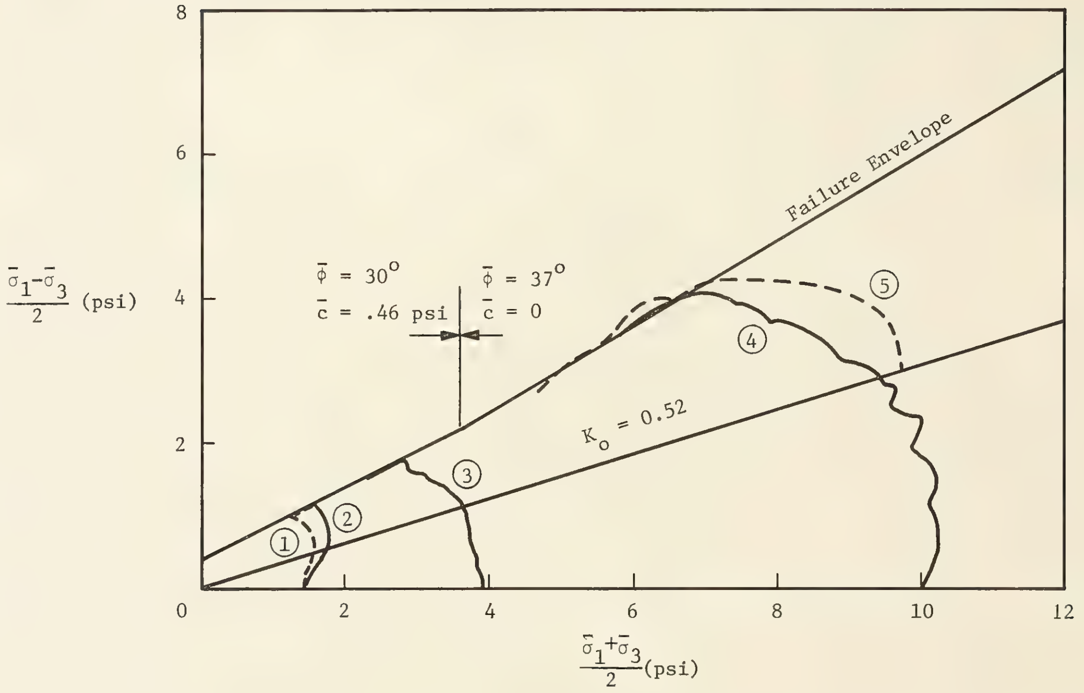

Figure 8. Stress path for triaxial tests performed on pelagic clay samples (Box A).

drawn as two straight line segments which appear to best approximate the data. The upper portion of the failure envelope, if extended, passes through the origin while the lower portion does not. A failure envelope which passes through the origin is characteristic of a normally consolidated soil and a non-zero intercept is characteristic of an overconsolidated soil. Once again it may be seen that the soil behaves in an overconsolidated manner at low consolidation stresses and in a normally consolidated manner at higher stresses. These results are similar to those obtained from the one-dimensional consolidation tests.

The strength parameters expressed in effective stresses needed to evaluate long-term anchor holding capacity and footing bearing capacity are defined by the Mohr-Coulomb failure law (Lambe and Whitman, 1969, p. 139)

$$\tau_{ff} = \bar{c} + \bar{\sigma}_{ff} \tan \bar{\phi} \tag{2}$$

where τ_{ff} = shear strength

$\bar{\sigma}_{ff}$ = normal effective stress on failure plane

$\bar{c}, \bar{\phi}$ = cohesion intercept and friction angle, respectively, expressed in terms of effective stresses

The parameters $\bar{c}$ and $\bar{\phi}$ can be readily obtained from the failure envelope of a stress path diagram such as Figure 8 (Lambe and Whitman, 1969, p. 141). The parameters obtained were:

	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2} < 3.5$ psi	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2} > 3.5$ psi
$\bar{c}$	.46 psi	0 psi
$\bar{\phi}$	30^{o}	37^{o}

Each of the five triaxial tests was performed to satisfy a particular need; a discussion of these needs follows:

Test 1. The test was performed at very low strain rate (3 percent per day).

Test 2. The test was performed at a more rapid strain rate (4 percent per hour). The results of Tests 1 and 2 were compared, and it was decided that the differences were not sufficient to warrant performing tests at the slower rate. The remaining tests were performed at about 4 percent per hour.

Test 3. The test was performed to define the failure envelope in the zone of slight apparent overconsolidation.

Test 4. The test was performed to define the failure envelope in a clearly normally consolidated region.

Test 5. The test was performed to determine the influence of K_o consolidation on strength properties.

The primary objective of the triaxial testing was to develop an estimate of the strength distribution in these pelagic clays to a significant sub-bottom depth (100 ft). Since drained and undrained failure envelopes generally correspond (Lambe and Whitman, 1969, p. 427) if effective stresses are used, the parameters $\bar{c}$ and $\bar{\phi}$ derived above can be used directly in a limiting equilibrium problem. Long-term or drained anchor holding capacity and foundation bearing capacity can be readily calculated using existing procedures (e.g., Taylor and Lee, 1973). Obtaining an undrained strength distribution for analysis of short-term capacity is somewhat more difficult. The stress paths of Figure 8 can be used as a starting point since they determine several different undrained shear strengths resulting from different initial consolidation conditions. However, there are two problems: (1) the initial conditions of the four isotropically consolidated samples do not correspond to the anisotropic consolidation conditions in-situ and (2) the strengths correspond to discrete situations; a means for interpolating between the situations is needed.

The problem of K_o consolidation is approached by observing the results of Tests 4 and 5. Both involve the same order of magnitude of stresses but Test 4 was consolidated isotropically while Test 5 was consolidated under K_o conditions. It may be seen that the shapes of the curves as they approach the failure envelopes are similar. A means of using this similarity has been suggested previously (Taylor, 1948, p. 383): A point is located on a stress path resulting from isotropic consolidation which has the same stress conditions as would result from K_o consolidation. That is, $\bar{\sigma}_3$ is equal to $K_o\bar{\sigma}_1$. The stress path which would result from K_o consolidation is assumed to be identical to the portion of the stress path resulting from isotropic consolidation between that point and the failure envelope. This procedure has been criticized (Ladd, 1965) because the paths are not usually identical. For the purposes of this report, the similarity between the two paths appears good enough to assume that they are identical.

To use the Taylor (1948) technique it is necessary to have an estimate of the parameter K_o for a variety of stresses. The value of K_o obtained from the one anisotropic consolidation test was 0.52, and will be assumed to apply to the entire range of stress. This may be a poor assumption for the low stress region; however, trials of other values did not significantly alter the resulting strength distribution. A plot of the line representing K_o equal to 0.52 is given on Figure 8.

The stress paths of interest are those above the K_o line in Figure 8. It is possible to interpolate between these paths by using a modified version of an equation given by Ladd (1965):

$$\frac{S_u}{\bar{\sigma}_{vo}} = \frac{\left|K_o + \bar{A}_f(1 - K_o)\right| \sin\bar{\phi} + \frac{\bar{c}}{\bar{\sigma}_{vo}} \cos\bar{\phi}}{1 + (2\bar{A}_f - 1)\sin\bar{\phi}} \tag{3}$$

where S_u = undrained shear strength

$\bar{\sigma}_{vo}$ = vertical effective stress during K_o consolidation (equal to field overburden pressure)

$\bar{A}_f$ = a pore pressure parameter

$\bar{c}$, $\bar{\phi}$, and K_o are as defined previously

To use this equation it is necessary to determine a distribution for the pore pressure parameter, A_f (defined by Skempton, 1954). For the NCEL tests, A_f is equal to the change in pore water pressure divided by the change in the difference between $\bar{\sigma}_1$ and $\bar{\sigma}_3$ during shear. The parameter A_f was obtained from each of the sets of triaxial test results, considering only the portions of the stress paths above the K_o line. In Figure 9 A_f is plotted versus the vertical effective stress following K_o consolidation. A line has been constructed between the data points for use in interpolation. In the clearly normally consolidated region, A_f is independent of $\bar{\sigma}_{vo}$ (Ladd, 1964, Figure III-18). In the pseudo-overconsolidated region a linear interpolation between the data points has been constructed.

An estimated undrained shear strength profile can be constructed using the data which have been developed to this point. The strength, S_u, as a function of overburden pressure, $\bar{\sigma}_{vo}$, is obtained by inserting the previously obtained values of $\bar{c}$, $\bar{\phi}$, K_o, and A_f into Equation 3. The overburden pressure as a function of sub-bottom depth is given by Figure 6. It is possible, therefore, to obtain shear strength as a function of sub-bottom depth. This strength profile is given by Figure 10.

This profile appears to be the best achievable with the present information. Most of the assumptions made in its derivation are relatively insignificant. Two important assumptions are that (1) the basic soil type remains constant with depth and (2) consolidation in a triaxial cell simulates field consolidation. It is planned to check the validity of these assumptions by obtaining long piston cores and determining the variation of water content and index properties with depth. If the assumptions of this report are correct, the index properties will be nearly constant and the water content will vary in a consistent manner predicable by consolidation testing.

For short-term use the strength profile of Figure 10 is recommended for such endeavors as direct embedment anchor analysis. It should be noted that several characteristics of the profile are not conducive to

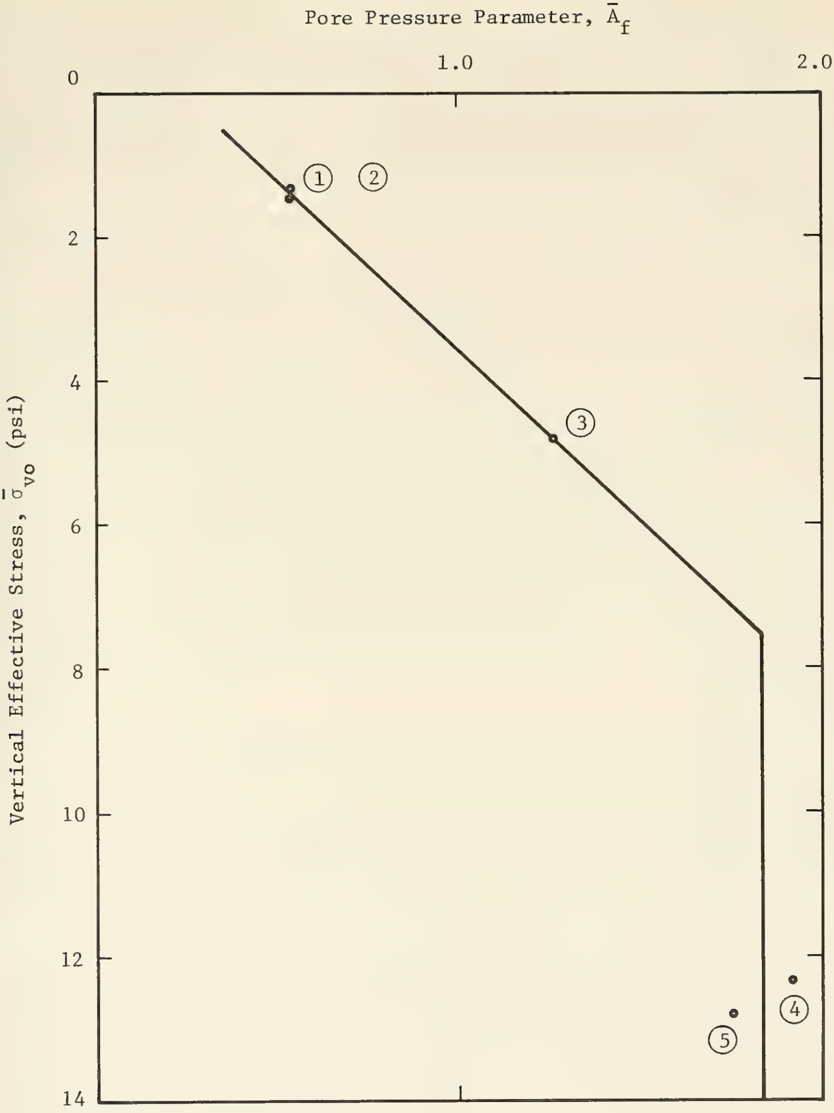

Figure 9. Pore pressure parameter, A_f, as a function of vertical effective stress - determined from triaxial compression tests (Box A).

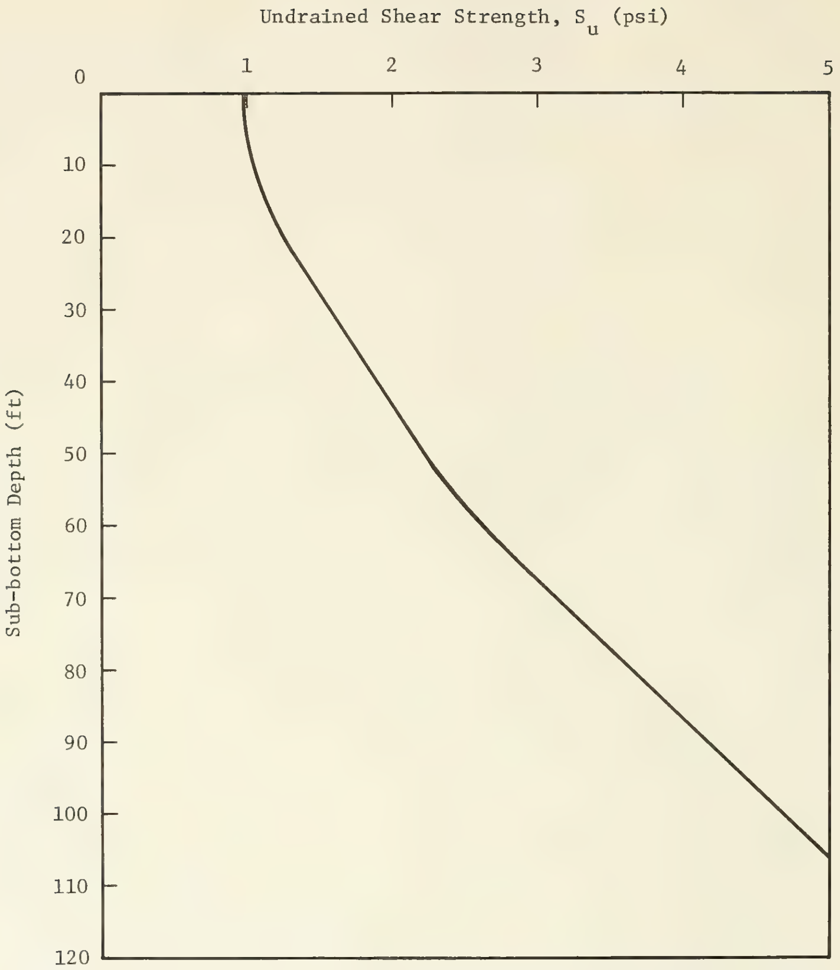

Figure 10. Estimated undrained shear strength profile for pelagic clay.

the development of high holding capacities. The relatively high strength soil near the surface can consume a large part of a penetrating anchor's kinetic energy. Unfortunately, since the strength increases very slowly with depth, a great penetration is needed to obtain even a moderate holding capacity. It may be necessary to downgrade the rated holding capacities of direct embedment anchors when they are being used in pelagic clays.

Correlation with Index Properties

One of the major goals of the current research is to derive correlations between index and engineering properties so that designs can be developed on the basis of tests on disturbed samples. Much of the data of this report will eventually be incorporated into such correlations and presented in a final report. For current purposes, however, only one correlation was tried. Lambe and Whitman (1969, p. 452) present a correlation between the ratio of undrained shear strength to overburden pressures ($S_u/\bar{\sigma}_{vo}$) and the plasticity index for marine clays. An estimate of this ratio was obtained using the triaxial test results, and Equation (3) and was found to correlate with the plasticity index in almost exactly the same manner (within 5 percent) as the Lambe and Whitman plot. However, this correlation was found to be valid for only the clearly normally consolidated region (sub-bottom depths greater than 50 feet). For more shallow embedments, a more complex relationship is needed. The ratio of strength to overburden pressure for depths greater than 50 feet was found to be about 0.32, a typical value for cohesive soils.

CONCLUSIONS

1. The index and engineering properties of two samples of pelagic clay obtained from two difference locations were almost identical.

2. The pelagic clay samples obtained from near the water-soil interface display characteristics of overconsolidation. That is, the properties are similar to those of material that has been preloaded at some time in the past.

3. As a result of the apparent overconsolidation, the soil is almost incompressible up to the apparent "maximum past pressure" of about 4 psi. The soil is very compressible beyond this point. Structural loads in excess of this level should be avoided.

4. The short-term strength of the pelagic clay samples from near the soil-water interface is relatively high (about 1 psi). This value is higher than previously obtained strengths of more disturbed cores.

5. These pelagic clay specimens were sensitive with the strength being reduced by as much as a factor of 6 during remolding. Care is required to prevent remolding during the installation of a structure.

6. A distribution of short-term shear strength with depth can be estimated through triaxial testing. The profile obtained from tests on these pelagic clay samples indicates a very gradual increase of strength with sub-bottom depth.

7. The relatively high strength near the surface may retard the penetration of direct penetrating anchors. However, since the strength increases very gradually with depth, large penetrations are needed to achieve a substantial holding capacity. It may be necessary to downgrade the rated holding capacity of direct embedment anchors when used in pelagic clay.

ACKNOWLEDGMENTS

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NOMENCLATURE

$\bar{A}_f$	Pore pressure parameter at failure
$\bar{c}$	Cohesion intercept expressed in terms of effective stress
c_v	Coefficient of consolidation
C_α	Coefficient of secondary compression
e	Sample void ratio
K_o	Coefficient of lateral earth pressure at rest
S_u	Undrained shear strength
u	Pore water pressure
σ	Normal stress
$\bar{\sigma}$	Effective normal stress
$\bar{\sigma}_1$	Major principal effective normal stress
$\bar{\sigma}_3$	Minor principal effective normal stress
$\bar{\sigma}_{ff}$	Normal stress on failure plane
$\bar{\sigma}_v$	Vertical effective stress
$\bar{\sigma}_{vo}$	Vertical effective stress in-situ
τ_{ff}	Shear stress on failure plane
$\bar{\phi}$	Friction angle expressed in terms of effective stresses

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13. ABSTRACT

Pelagic clay is a common ocean soil type covering over 30 percent of the seafloor. Two high quality box core samples of pelagic clay were obtained and subjected to index and engineering property testing. The results of tests on both cores were similar indicating little areal variability. The shear strength near the soil-water interface was about 1 psi, a relatively high value, and the soil was found to be virtually incompressible up to a compressive stress of about 4 psi. When remolded, the strength of the soil was significantly decreased, by as much as a factor of 6. The soil was also found to be very compressible beyond a stress of 4 psi. A technique was developed for using triaxial test results to derive shear strengths below the level of sampling. The resulting strength profile indicates a very gradual increase of strength with depth. Implications regarding the use of direct embedment anchors are discussed.

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Clay soils						
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Pelagic zone						
Shear strength						
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Soil properties						

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